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Development of Computer Simulation Model for Urban Region Using Xp-Swmm in Savannah, Georgia

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Development of Computer Simulation Model for Urban Region Using XP-SWMM in Savannah, Georgia

by

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DEDICATION

To Lacey, Gunner and Gracey. You make this all worthwhile.

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I would like to thank my wife and son for being there throughout this entire journey. I would also like to thank several of the employees of HGBD, Inc. in Savannah, Georgia – Jeff, Billy Ray, and Steve – for their help and encouragement.

I would also like to thank Dr. Jon Goodall and Dr. Michael Meadows for their guidance and direction throughout the duration of this project. I would also like to thank Drs. Jasim Imran and Enrica Viparelli for taking the time to serve on my committee.

"It's been a long journey, but I have been blessed."

- Ricky Atkinson

ABSTRACT

The objective of this thesis was to determine if it is possible to create, calibrate, and validate a computer simulation model given a limited amount of measured data. In June 1999, significant flooding was experienced throughout the Casey Canal North drainage basin in Savannah, Georgia. Time-depth rainfall data from a single gage was recorded for this event, along with peak water surface elevations throughout the basin. The computer model chosen for this application was XP-SWMM, which was approved by the Federal Emergency Management Agency to simulate both one dimensional and two dimensional hydraulic models. XP-SWMM was chosen due to its ability to dynamically link the subsurface drainage system with the overland flow experienced during significant rainfall events. The measured peak water surface elevation data was divided into two, with one dataset used to calibrate the model and the second used for model verification. The model calibration was completed by manually adjusting certain hydrologic model parameters within an acceptable range in order to match field observed peak water surface elevations. The model evaluation showed that the peak water surface elevations estimated by the model matched the levels observed in the field and that inundated road intersections observed during the flooding event were correctly predicted by the model.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Flooding is a significant problem that impacts regions across the United States. According to the Federal Emergency Management Agency (FEMA), the average annual flood loss in the United States is $2.7 billion dollars (Resources, 2012). Since 1978, FEMA has paid nearly $33 billion in flood damages caused by significant events, which are defined as a flooding event that has at least 1,500 paid losses, or claims (FEMA, 2011). In general, floods are one of the most common hazards experienced in the United States (NFIP, 2002). The effects of flooding range from localized flooding within a neighborhood to large-scale riverine flooding that can impact large regions that sometimes cross multiple states.

Flooding can occur in several different ways and civil engineers have developed a variety of computer models for simulating different flooding scenarios. Flooding can occur in coastal areas due to storm surge. In other cases, floods can develop slowly, sometimes over a period of days within large river basins (e.g., the 2011 flood along the Mississippi River). Flash floods, the subject of this research, are different in that they develop quickly, sometimes in just a few minutes, but can still have large impacts on more localized areas. In all three cases, flooding often occurs when engineering infrastructure (e.g., levees or stormwater infrastructure) fail to mitigate the impacts of severe rainfalls. Flooding within urban districts, such as the City of Savannah, can be a particularly complex issue because of hydrologically modified landscapes that consist of impervious surfaces and stormwater infrastructure. Flooding in urban regions often occurs due to poorly functioning stormwater infrastructure that results in drainage congestion and overbank flow of open channels during significant rainfall events (Dey et al., 2010).

In order to accurately predict the severity of flooding within urban areas, engineers use a variety of stormwater models to analyze the drainage systems. One problem experienced by many municipalities is a lack of historical flood data to use to calibrate and validate stormwater models. Another major problem facing many municipalities today is the financing of major stormwater projects (Guidance, 2006). Under the current requirements set forth by the Environmental Protection Agency in regards to pollutant discharge through stormwater, stormwater quality and treatment are necessitating increased focus by many municipalities. Typically, most municipalities are expected to cover all necessary treatment costs associated with flood control and treatment (Guidance, 2006).

The focus of this thesis was on these flooding issues, specifically as they related to an individual flooding event in the downtown area of Savannah, GA (Figure 1.1). The storm of focus occurred on June 29, 1999 when Savannah experienced a rainfall event that caused extensive flooding throughout the downtown area. A single rainfall gage within the watershed recorded over twelve inches of rainfall during an eleven hour time period. The goal of this thesis was to create, calibrate, and validate a computer simulation model of this flooding event using limited measured data and determine the potential suitability for expanded use of this model.

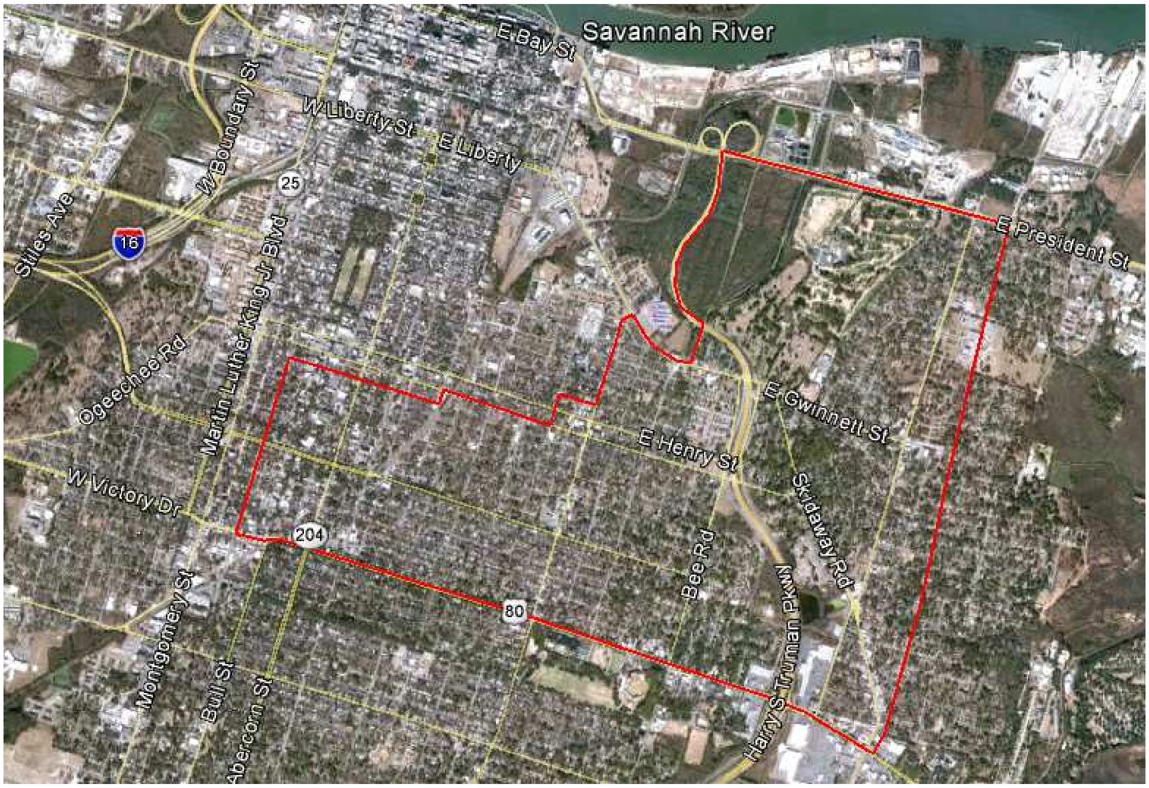

Figure 1.1 Overall Drainage Area Map

1.2 Background

Savannah is located in southeast Georgia, approximately twelve and one-half miles inland from the Atlantic Ocean along the Savannah River. The city lies within the Coastal Plain physiographic region with ground elevations typically ranging from zero to 25 feet above mean sea level. According to the 2010 census, the City of Savannah has a population of over 136,000 people. With the inclusion of the surrounding areas, the Savannah metropolitan area has a population of over 347,000 people.

Savannah experiences, on average, over 47 inches of rainfall in a calendar year. The majority of rainfall occurs during the months of June, July, August, and September (Figure 1.2).

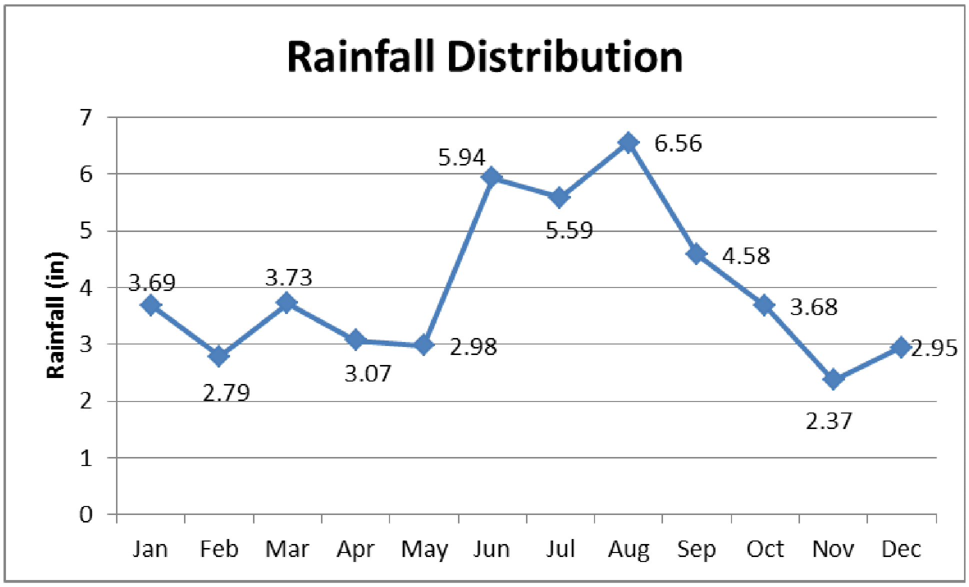

Figure 1.2 Average Annual Rainfall Distribution for Savannah, Georgia (NOAA, 2011)

In an effort to better understand the flooding risks within the municipal boundary, the City commissioned flood studies to be performed by local engineering groups. One such area of concern was the Casey Canal North basin. This watershed is roughly 2,000 acres in size, of which forty percent of the total land area is impervious. Within the Casey Canal North basin, there is a significant wetland system which serves as the primary drainage outfall.

1.3 Study Objectives

The objectives for this thesis were to:

- Use existing data (e.g., digital topographical, storm water management system data, and meteorological data) to create a computer model of the existing storm water management system within the Casey Canal North Basin.
- Calibrate the model so that it is able to simulate flood inundation due to the rainfall event on June 29, 1999.
- Compare the calibrated results to independent field data gathered about flood inundation levels to evaluate the accuracy of the model.
- Examine the viability of using a single storm event to accurately predict future flood elevations.

1.4 Thesis Organization

The thesis is organized into six chapters. Chapter 2 is a literature review of past work examining urban flooding issues focusing in particular on the process of modeling urban flooding. Chapter 3 presents the methodology used to construct and evaluate the urban flooding model. Chapter 4 portrays model results along with a discussion of the model calibration and evaluation procedure. Finally, Chapter 5 offers concluding remarks and a discussion of possible limitations using this approach and future work that can be completed based upon the model resulting from this thesis.

CHAPTER 2

LITERATURE REVIEW

2.1 Background on Computer Programs to Model Flooding

There are several different software programs commercially available to model flooding. The most common method for modeling such systems is one dimensional modeling such as HEC-RAS (USCOE, 2008). Although the use of one dimensional models has been the industry standard for some time, several limitations are associated with this form of modeling, including the inability to properly represent the river bathymetry, the inability to model large scale extreme events, and the inability to model very complex systems (anastomosing rivers) (Merwade et al., 2008). Another limitation of one dimensional models is the use of a box finite difference scheme. For instance, in situations where the flow transitions from subcritical to supercritical, the calculations are impaired due to each condition requiring different algorithms. Neal et al. (2012) showed that the results were reasonably consistent between one dimensional and two dimensional models, as long as the flows vary gradually and the model time steps were selected appropriately.

2.2 Review of the XP-SWMM Simulation Model

As computer hardware has improved and subsequently caused a reduction in overall computational time of numerical simulation models, two dimensional computer modeling has become a more viable solution. In this study, a two dimensional modeling program, XP-SWMM, was selected as the modeling software. XP-SWMM is based upon the Environmental Protection Agency's SWMM computational engine. Barco et al. (2008) defines SWMM as a "dynamic rainfall-runoff model for simulation of quantity and quality problems associated with runoff from urban areas." SWMM can be used in a variety of applications from urban drainage to flood routing. In May 2010, FEMA classified XP-SWMM as approved for use in the NFIP in one-dimensional and two-dimensional modeling (Numerical, 2011).

According to FEMA, XP-SWMM is capable of considering the loss of floodplain storage and the corresponding loss of conveyance while one-dimensional steady flow models are incapable of doing so (Floodway, 2001). By using a two-dimensional model such as XP-SWMM, this interaction will give more accurate predictions of flood wave propagation than one-dimensional models. By using the one-dimensional water surface elevation profile as a boundary condition for the two-dimensional simulation, XP-SWMM maintains a dynamic link between the two simulations (Numerical, 2011). Seyoum et al. (2011) also found that a coupled 1D/2D model, such as XP-SWMM, is capable of reproducing the interaction between drainage system flow and surface flow. This capability is suited to modeling the interaction between surcharged drainage inlets and excess surface flow through roadway gutter and other overland flow pathways which are common in urban areas.

2.3 Review of Methodology for Modeling Hydrologic Systems

Any time a hydrology modeling project is undertaken, several steps must be taken to create the model. Typically, modeling consists of four steps: model set-up, model calibration, model validation, and exploitation (Vidal et al., 2007). This process has been criticized due to the method of validation. Models are typically calibrated based upon an agreement between modeled results and some measured data in the field, typically without any consideration of the physical situation. For example, during large rainfall events, it is likely that there will be a significant quantity of debris that is transported by the runoff. In certain areas, this could account for blockage of pipes or drainage inlets and, as a result, inundation in a particular location. However, most storm water models ignore blockage of pipes and drainage inlets. Vidal et al. (2007) used a methodology to show that calibration is merely one step for an overall assessment for a model. This methodology defines calibration as “the procedure of adjustment of parameter values of a model to reproduce the response of reality within the range of accuracy specified in the performance criteria”, which is subsequently defined as the “level of acceptable agreement between model and reality.”

Calibration is a crucial step to creating a watershed model (Knebl et al., 2005). There are many different methodologies used to calibrate watershed models. Barco et al. (2008) shows calibration of the SWMM model using the Box complex method. In the Box complex method, the calibration parameters (called vertexes) are selected and entered into the model. The computations are evaluated in accordance with the objective function and the vertex with the greatest function value is rejected. The remaining vertexes are then averaged and a new vertex defined. This process continues until the termination criterion has been satisfied. This is generally completed within an automatic calibration process. Even though this calibration procedure is extensive, use of the procedure does not guarantee that the user will find a global optimum (Barco et al., 2008).

James (2002) surmised that it is reasonable to apply universal corrections to variables that have been systematically derived. For example, if the modeled results indicate that the peak water surface elevations are below the field measured values, it is reasonable to adjust curve numbers for all sub-watersheds given that the curve numbers were similarly derived. This allows for mass adjustment and prevents critique of each sub-watershed value. The ability to mass adjust certain variables that have been systematically derived deem it possible to calibrate urban models which tend to be large and extremely complex (James, 2002). This principle where calibration parameters were altered in groups according to the relationship of the modeled results to the field measurements was applied in this thesis, as described in the methodology section.

Many different parameters are used for calibration in a watershed models including curve number, time of concentration, drainage areas, overall slopes, UH Peak Rate Factor, and Manning's roughness coefficients. In Barco et al. (2008), four calibration parameters were selected: imperviousness percentage, width, impervious depression storage coefficient, and channel Manning's roughness coefficient. In Ogden (2011), impervious percentage, drainage density and drainage widths were used for calibration. James (2002) shows that calibration of urban models generally requires calibration of at least the peak flow first and the entire hydrograph shape.

In situations of heavy rainfall, the most important parameters for calibration of urban hydrology models are overall slopes and drainage areas, not necessarily infiltration rates and impervious percentages (James, 2002). This is also verified by Ogden (2011), which states “in the case of moderate to extreme rainfall events, model sensitivity to heterogeneous parameters is diminished, which enables use of event-based calibration.” Several calibration parameters may be used simultaneously for watershed modeling. Generally, runoff within urbanized catchments that have significant quantities of impervious area has been shown to be more sensitive to the rainfall rate than other factors (Ogden et al., 2011).

2.4 Summary of Literature Review

The goal of this thesis, that is to create an accurate model of flood inundation resulting from the June 29, 2012 flood event in Savannah, GA, was completed in light of this past research in storm water modeling. In particular, the past results from model calibration and evaluation were used to guide the methodology development, as described in the following section.

CHAPTER 3

METHODOLOGY

3.1 XP-SWMM Overview

XP-SWMM was selected as the model for this thesis. XP-SWMM is based upon the Environmental Protection Agency's SWMM computational engine and was selected for several reasons. (i) XP-SWMM has the capabilities to dynamically link the one-dimensional model and two-dimensional simulation. (ii) XP-SWMM is also capable of modeling backwater effects, flow reversal, surcharging, pressurized flow, tidal outfalls and interconnected ponds. (iii) XP-SWMM is capable of considering the loss of floodplain storage and the corresponding loss of conveyance while one-dimensional steady flow models are incapable of doing so (Floodway, 2001). By using a two-dimensional model such as XP-SWMM, this interaction gives more accurate predictions of flood wave propagation than one-dimensional models.

This methodology section is organized following the packages in XP-SWMM that were used in the Savannah simulation. First the hydrology portion of the simulation is presented as it relates to the RUNOFF package. Second the hydraulics portion of the simulation is presented as it relates to the EXTRAN package. Third, the water surface modeling is presented as it relates to the TUFLOW package. Finally, the input data for the simulation, focusing in particular on the rainfall data used to drive the model and the peak water surface elevation (WSE) data used to evaluate the model. Figure 3.1 below details the input parameters and output results of each module within the program.

3.2 Hydrologic Simulation using the RUNOFF Package

The RUNOFF hydrologic module within SWMM was used to create the hydrologic model for the Casey Canal North Basin, and to simulate the quantity of runoff within the basin using a pre-defined rainfall hyetograph (James, 2001). In the case of the calibration model, the recorded rainfall depths from the July 1999 storm was used as discussed previously. The calculation method selected for this study was the Soil Conservation Service Technical Release 55 (TR-55) methodology. TR-55 is a simplified method for calculating runoff from rainfall events in urban environments. According to the USDA: "The model described in TR-55 begins with a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed" (USDA 1986). Several key variables are required to predict runoff volume using TR-55: Rainfall, Drainage Area, Impervious Percentage, Curve Number, and Time of Concentration.

3.2.1 Drainage Area

The Casey Canal North basin covers approximately 2,000 total acres. To determine the drainage area for each sub-basin within the watershed, topographic maps were used to define ridges and low-lying areas. In the mid-1990s, a topographic survey was conducted for the City of Savannah which provided a grid survey along the centerline of the roadways. This information, along with LIDAR topography provided by the City of Savannah was used to delineate the drainage areas for each sub-basin. As a general guideline, most intersections within the well established downtown area of Savannah are the low points of their respective sub-basin. Storm drainage inlets are located on each corner of the intersection with a single manhole (junction box) located in the center of the intersection. For the purpose of this study, the sub-basin for each inlet within the intersection was combined and a single node was established at the middle of the intersection. A sample of the drainage area delineation is given below in Figure 3.1. A complete drainage area map is attached in the Appendix. Tables are provided in the Appendix with a complete listing of drainage area for each sub-basin.

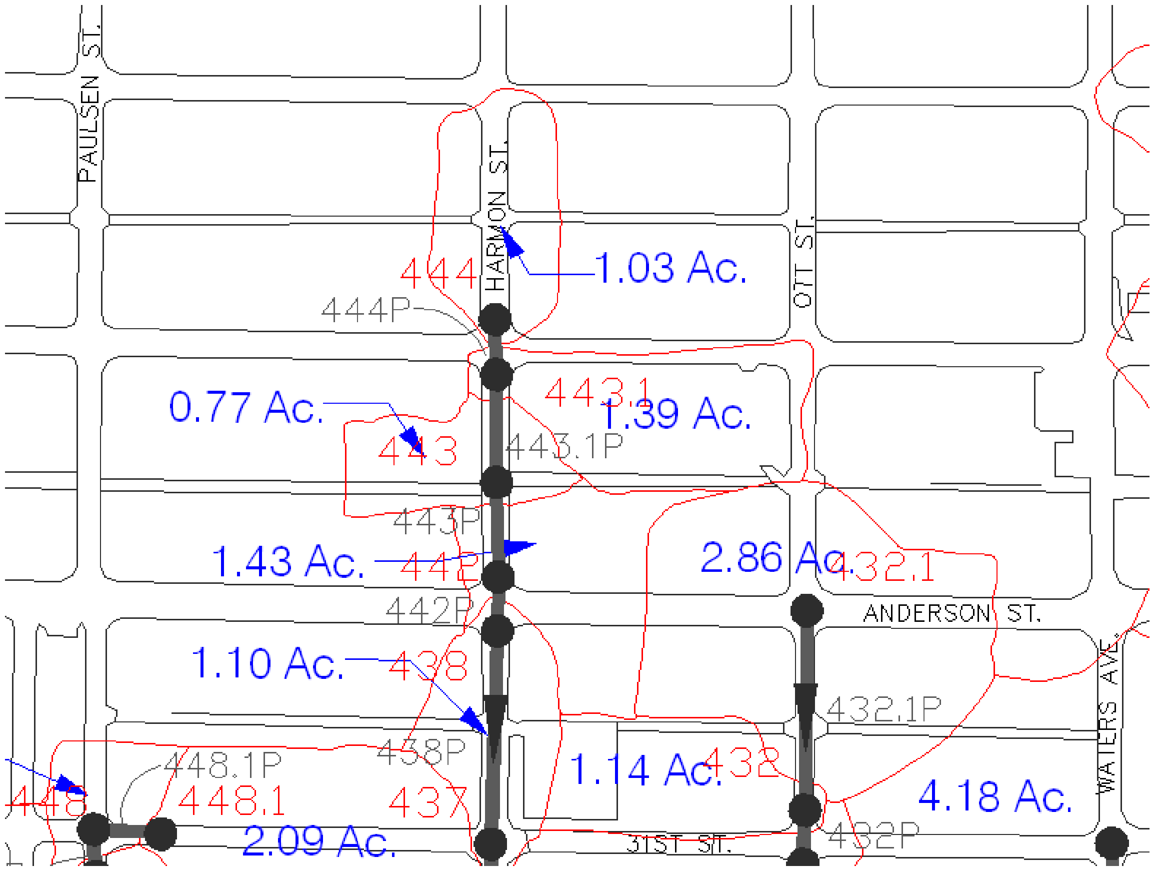

Figure 3.1 Typical sub-watershed drainage area delineation for the study region. Red lines represent the watershed boundaries; Blue text denotes the watershed area; and existing stormwater infrastructure is represented by the black circles as inlets and lines with flow arrows as pipes.

3.2.2 Impervious Percentage

In most urbanized regions, the majority of impervious area is composed of streets, parking lots, and other transportation-related structures. According to the TR-55 manual, an increase in impervious area (urbanization) has a significant impact on the infiltration rate of soil (1986). It has been determined that over ninety percent of rainfall which falls on impervious area (asphalt, concrete, etc.) is converted to runoff (Beckwith et al, 2007). Other studies have found that one acre of paved parking will generate over sixteen times the amount of runoff than a pasture of the same size (OEC, 2012).

It was assumed for this report that all impervious areas were directly connected to the storm drainage system. In order to determine the impervious percentage for each sub-basin, aerial topography maps were created for the watershed and impervious areas were delineated for each sub-basin. As previously discussed, the majority of the Casey Canal North drainage basin lies within an urbanized district. The impervious percentages within the watershed typically ranged from sixty to eighty percent. A complete listing of impervious percentage is provided in the Appendix. The impervious area within a watershed is necessary to establish a composite curve number.

3.2.3 Curve Number

Curve numbers are assigned for each drainage area within the watershed. Curve numbers are indicators of the runoff potential of a watershed during a rainfall event. Several variables influence the pervious area curve number for a watershed: hydrologic soil group, cover type, treatment, hydrologic condition and antecedent runoff condition (USDA, 1986). According to sources, the most important variables for defining a curve number are the hydrologic soil group and cover type (SCS, 2011). In order to establish the pervious soil curve number, the hydrologic soil group for each drainage area was determined from the Soil Conservation Service map for Bryan and Chatham Counties. Next, the existing cover type and treatment was determined from aerial topography provided by the City of Savannah. The soils within the study area are classified as hydrologic groups B/C. Generally, most open area within the watershed is tree covered. The general consensus for the pervious area curve number chosen for this watershed was 65, which accounts for a wooded area with fair to good ground cover. Once the curve number was established for each drainage area, the composite curve number was calculated for each drainage area based on the pervious area curve number and impervious percentage using the following formula (NEH, 2004)

$$CN_c = CN_p + \left(\frac{P_{imp}}{100}\right)(98 - CN_p) \qquad 3.1$$

where CN_c is the composite Curve Number, CN_p is the pervious area Curve Number, and P_{imp} is the percent imperviousness. A complete listing of the composite curve numbers is included in the appendix.

3.2.4 Time of Concentration

Time of concentration (T_c) is a critical parameter of the TR-55 methodology. Time of concentration is defined as "the time it takes for runoff to travel to a point of interest from the hydraulically most distant point" (USDA, 1986). Several key factors are important to consider when calculating the time of concentration: surface roughness, channel shape and flow patterns, and slope. In urbanized areas, such as the Casey Canal North Basin, these three factors are drastically modified when compared to pre-development conditions. Subsequently, the time of concentration is reduced by the following conditions: surface roughness is typically greatly decreased due to a reduction in the retardance to flow; channel slope and flow patterns are changed by reducing the flow lengths; and the slope of the watershed is typically altered during development as channels are straightened (Iowa, 2008).

In spite of the importance of time of concentration, it is sometimes very difficult to determine. There are several different methodologies for calculating time of concentration; however, the most common form, and the method used for this study, is the NRCS Velocity Method. In this method, time of concentration is the summation of the respective travel times (T_t) for the different methods of flow: sheet flow, shallow concentrated flow, and open channel flow (USDA, 1986).

Sheet flow occurs as water flows across plane surfaces, typically near the beginning of stream formation. On average, sheet flow depth is approximately one-tenth of a foot (USDA, 1986). Studies have shown that the maximum length of sheet flow is limited to 100 feet. According to the NRCS Velocity Method, the travel time for sheet flow is calculated as

$$T_t = \frac{0.007[(n)(L)]^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad 3.2$$

where T_t is the travel time in hours, n is Manning's roughness coefficient, L is the flow length in feet, P_2 is the 2-year, 24-hour rainfall in inches, and S is the slope of hydraulic grade line in ft/ft.

Once sheet flow has occurred for approximately 100 feet and the depth has exceeded one-tenth of a foot, shallow concentrated flow begins. The travel time for shallow concentrated flow is calculated as

$$T_t = \frac{L}{3600V} \qquad 3.3$$

where T_t is the travel time in hour, L is the flow length in feet, and V is the average velocity in feet per second. The average velocity for this equation can be found empirically by using a graph in the TR-55 manual, page 3-2, or as

$$\text{Unpaved: } V = 16.1345(s)^{0.5} \qquad 3.4$$

$$\text{Paved: } V = 20.3282(s)^{0.5} \qquad 3.5$$

where s = watercourse slope in feet/feet (Iowa, 2008).

Open channel flow occurs where shallow concentrated flow enters a storm conveyance system (ie pipes, culverts, ditches, canals, etc.) The travel time during open channel flow is calculated using Equation 3-4 with the average velocity calculation following Manning's equation

$$V = \frac{1.49 R^{\frac{2}{3}} S^{\frac{1}{2}}}{n} \qquad 3.6$$

where V is the average velocity in feet per second, R is the hydraulic radius in feet, S is the slope of the hydraulic grade line in feet/feet, and n is Manning's roughness coefficient.

For the purposes of this study, the time of concentration was calculated and then input into the model. As part of the calibration of the model (as discussed later in this section) the time of concentration was one of the variables adjusted to simulate the measured results. The final values are summarized in the Appendix.

3.3 Hydraulics using the EXTRAN Package

XP-SWMM is capable of modeling backwater effects, flow reversal, surcharging, pressurized flow, tidal outfalls and interconnected ponds. Within XP-SWMM, the module used for the computations was EXTRAN. EXTRAN is specifically used to route inlet hydrographs through the network of pipes and junctions to the outfalls of the system. A schematic of the process completed by EXTRAN is presented in Figure 3.2.

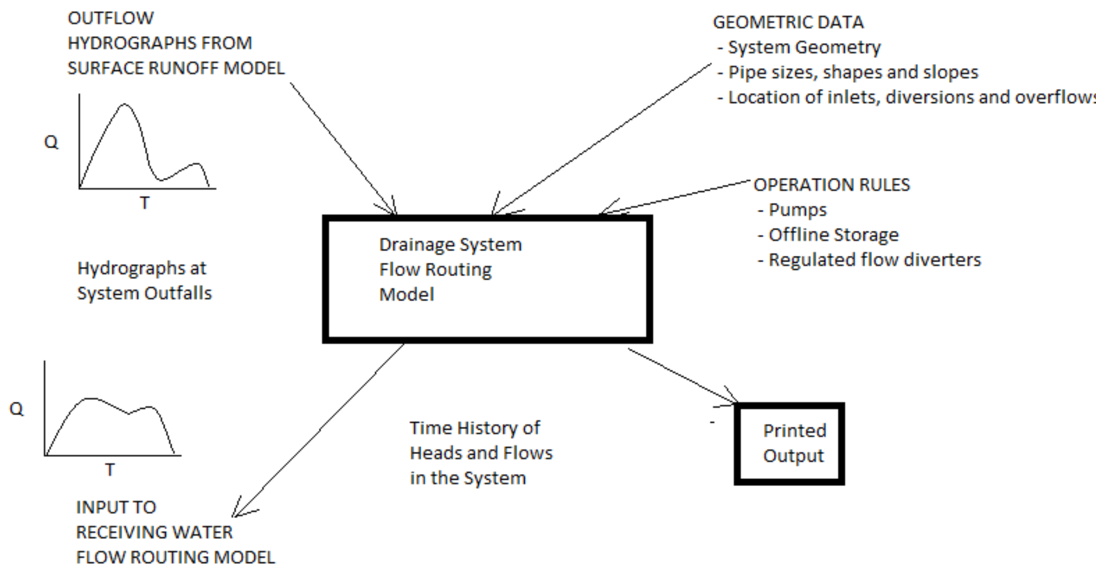

Figure 3.2 Schematic representation of EXTRAN process (James, 2000).

XP-SWMM solves the complete St. Venant equations for gradually varied, one dimensional, unsteady flow. Within EXTRAN, the momentum equation is combined with the continuity equation to yield

$$\frac{\partial Q}{\partial t} + gAS_f - 2V\frac{\partial A}{\partial t} - V^2\frac{\partial A}{\partial t} + gA\frac{\partial H}{\partial x} = 0 \qquad 3.7$$

where Q is the discharge along the conduit, V is the velocity in the conduit, A is the cross-sectional area of the flow, H is the hydraulic head, and S_f is the friction slope (James, 2000).

Within the hydraulic module of XP-SWMM, inlets, manholes, storage areas, and junction boxes are defined as "nodes." Conduits, channels, ditches, and weirs are defined within the model as "links." Several key components are necessary for the calculations of the EXTRAN module: runoff hydrographs generated by the RUNOFF module, link and node geometry, outfall conditions, and pump characteristics.

3.3.1 Link and Node Geometry

To define a link within XP-SWMM, several factors are required: link shape, upstream and downstream invert elevations, Manning's roughness coefficient, and diameter/height. To define these objects, the storm drainage system database provided by the City of Savannah was used. One significant limitation of the database is the lack of information on pipe materials for determining the Manning's roughness coefficient on conduits. The pipe materials within the drainage basin range from concrete, metal, high-density polyethylene to hand-laid brick. Due to the sheer volume of pipes within the system, a standard value of n was assumed to be 0.012.

To define a node within XP-SWMM, several factors are required: inlet hydrograph, invert elevations, off-line storage areas, and surface elevation. The inlet hydrographs are created during the modeling of the RUNOFF module. The hydrographs act as the source of excess runoff that is input into the EXTRAN module. To define the invert elevations, the storm drainage system database from the City of Savannah was used as previously discussed. Within the Casey Canal North basin, there are several low-lying wetlands which serve as off-line storage along the Kayton Canal. The storage areas and corresponding elevations were calculated and then entered into the respective nodes. To determine the surface elevations, the elevation at each node was taken from the digital terrain model (DTM). Tables are provided in the Appendix detailing the node and link geometric data that was entered into the model.

3.3.2 Outfall Conditions

The main outfall for the Casey Canal North basin is through the Kayton Canal. The Kayton Canal flows to a storm drainage pumping station located adjacent to the

Savannah River. This outfall location is located approximately 12.5 miles inland from the confluence of the Savannah River and the Atlantic Ocean; therefore, tidal elevations tend to have a significant impact on most streams in this region. However, there is a tide gate at this location which prevents any backwater effects of the changing tides. In addition, the storm water pumping station is designed such that the maximum outflow is preserved regardless of the tidal elevations. This allows for the Kayton Canal to maintain flow at all times during a rain event and prevents any backwater effects that would be caused by the changing tides.

3.3.3 Pump Characteristics

Currently, there are twelve separate pumps located at the Kayton Canal pump station. The maximum flow capacities for the pumps were entered into the model as follows:

Table 3.1 – Maximum flow capacities of pumps within drainage system.

Pump	Minimum Flow (cfs)	Maximum Flow (cfs)
1	67	67
2	67	67
3	107	107
4	54	107
5	107	107
6	107	107
7	107	107
8	54	107
9	54	107
10	107	107
11	0	0
12	0	0

3.4 Surface Modeling using the TUFLOW Package

Once the modeled water surface elevations exceeds the top of the drainage inlets, the excess water is routed onto the surface as defined in the model. Two-dimensional surface modeling is accomplished using the Two-dimensional Unsteady FLOW (TUFLOW) module within XP-SWMM. For free surface flow, TUFLOW solves the full two-dimensional, depth averaged, momentum and continuity equations

$$\frac{\partial \varsigma}{\partial t}+\frac{\partial (Hu)}{\partial x}+\frac{\partial (Hv)}{\partial y}=0$$

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}-c_f v+g\frac{\partial \varsigma}{\partial x}+gu\left(\frac{n^2}{H^{\frac{4}{3}}}+\frac{f_l}{2g\partial x}\right)\sqrt{u^2+v^2}-\mu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial x}=F_x$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}-c_f u+g\frac{\partial \varsigma}{\partial y}+gv\left(\frac{n^2}{H^{\frac{4}{3}}}+\frac{f_l}{2g\partial y}\right)\sqrt{u^2+v^2}-\mu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial y}=F_y$$

where ς is the water surface elevation, u is the depth averaged velocity component in x direction, v is the depth averaged velocity in y direction, H is the depth of water, t is the time, x is the distance in x direction, y is the distance in y direction, c_f is the Coriolis force coefficient, n is Manning's roughness coefficient, f_l is the form (energy) loss coefficient, μ is the horizontal diffusion of momentum coefficient, p is the atmospheric pressure, ρ is the density of water, F_x is the sum of components of external forces in x direction, and F_y is the sum of components of external forces in y direction.

In order to create the ground surface used in the model, a digital terrain model (DTM) was created in AutoCAD Civil 3D using the field run topography and the LIDAR topography provided by the City. Once the topographic information was entered into Civil3D, a project site area with elevations on a one hundred feet by one hundred feet grid was exported to an ASCII file. The ASCII file was then imported into XP-SWMM and a DTM was created. In order to create the two-dimensional flow surface, XP-SWMM created cells on the surface with an elevation assigned to the center point from the DTM. See Figure 3.3 below for a schematic representation of the cells versus the natural ground surface: (XP2D, 2011)

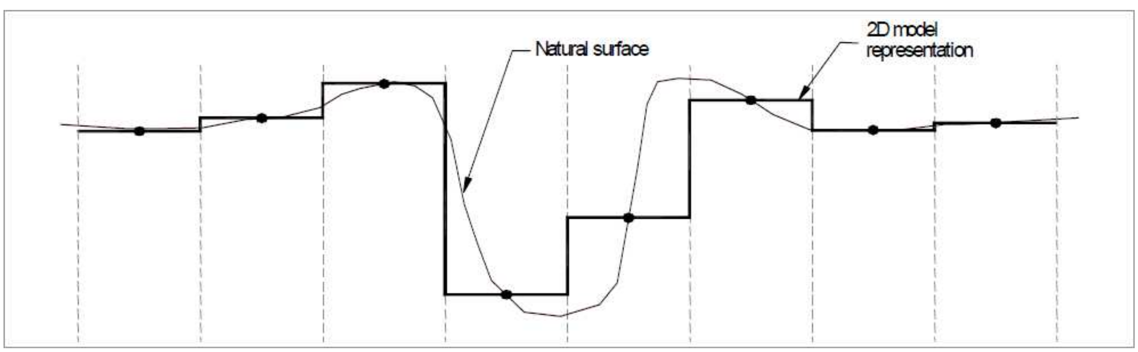

Figure 3.3 Graphic of 2D model representation of natural ground surface.

The software used for this study was limited to a maximum of 10,000 cells. Due to the large area of the basin, each cell was approximately ten meters square. While the number of available cells does limit the precision of the model, it was determined to be within the acceptable range for the purpose of the study. After the creation of the cells in XP-SWMM, each node was linked to the two-dimensional surface to establish the node top elevations (the ground surface). Once the calculated water surface elevation in the model reaches this elevation, surface storage will begin.

3.5 Model Calibration and Evaluation

On June 29, 1999, the City of Savannah experienced a significant rainfall event that caused extensive flooding within the Casey Canal North basin. Rainfall depth was recorded at a City rainfall gage near the Casey Canal North outfall. The data for this rainfall event was recorded cumulatively on an hourly basis. This data was subsequently input directly into the model. According to the records, 12.1 inches of rainfall was recorded over an eleven hour period (Figure 3.4).

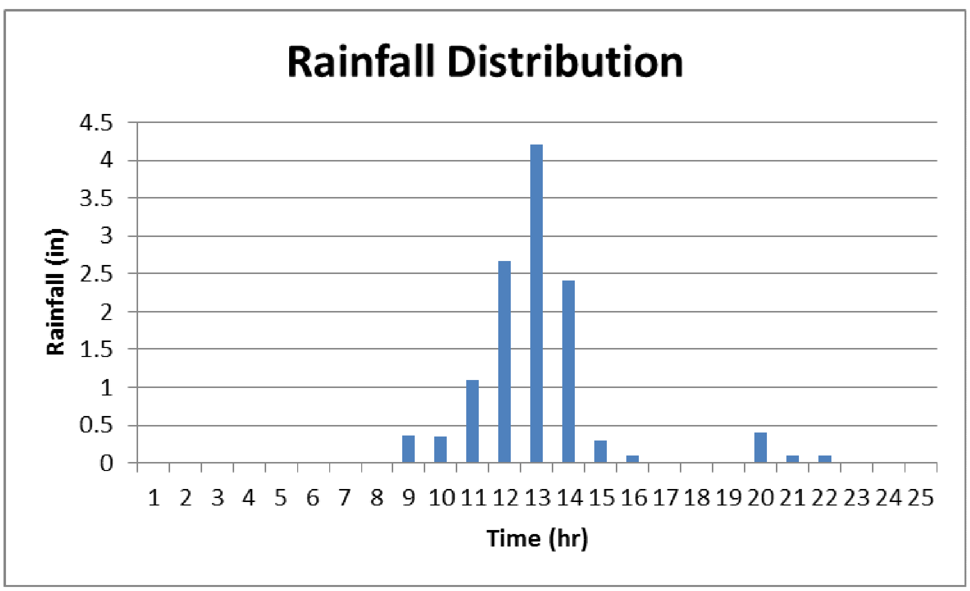

Figure 3.4 Rainfall hyetograph of June 29, 1999 storm.

3.6 Flooding Records

As is common practice in the City of Savannah, field surveys were done immediately following the storm to document all structural and roadway flooding. In addition, peak water surface elevations were obtained at several locations throughout the basin based on high water marks and eye witness accounts. These data points were plotted on an overall map of the basin. Of the 15 locations that were documented as structural flooding, five were ignored in this analysis. For four of the points, flooding was recorded; however, no elevations were established at the time. The flooding depth at the remaining location was recorded as excessive (over three feet) given the topography, and reports obtained from the residents immediately following the storm indicate that the flooding experienced was caused by the wave action of cars as they traveled down the submerged roadway. For the roadway intersections where flooding was noted, no elevations were established. The overall drainage basin was subdivided into three separate sections with several points lying within each section. These points were randomly sorted into two separate categories: a calibration set and an evaluation set.

For this thesis, the parameters for calibration were all part of the hydrologic portion of the model: curve number, time of concentration, and drainage area. No calibration was performed using parameters from the hydraulic portion of the model. Most notably, the Manning's roughness coefficients were selected using standard values and subsequently not modified. Even though Manning's roughness coefficient is generally used as a calibration parameter, Leandro et al. (2011) report ineffectiveness on the output results when using Manning's roughness as the calibration parameter and warn against using Manning's roughness to account for errors within models.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Results

4.1.1 Model Calibration

A test run of the storm of record was conducted using the default values for curve number, time of concentration, and drainage area as calculated above. Once the initial test run was completed, the peak water surface elevations were compared to the elevations recorded in the calibration data set. Based upon the results, modifications were made to curve numbers, time of concentration, and drainage area.

To modify the composite curve number, the impervious percentage of each respective drainage area was modified. Since the initial impervious percentage calculations were done using aerial photography, it is reasonable to surmise that there is a percentage of error associated with the initial measurements. By increasing the percentage of the impervious area, the amount of runoff generated within each sub-watershed increased. As the impervious percentage was modified, the aerial photographs were consulted to ensure that the impervious percentage was within reason.

The minimum time of concentration used within this thesis was ten minutes. To modify the time of concentration, adjustments were made in mass to adjust the time of concentration.

Minimum adjustments were made to the drainage areas of the sub-watersheds. Based upon the field topography provided, along with the LIDAR topography, the initial assumptions regarding drainage area proved to be reasonable.

Once the second test run was made, the peak water surface elevations were again compared to the calibration data set and modifications were again made to the calibration variables. Several additional calibration runs were made until the calculated peak water surface elevations closely resembled the recorded data set peak water surface elevations. After reaching this important step, the model was deemed acceptable to perform model evaluation. The final difference between the calculated and measured values for the calibration data set ranged from one-tenth to six-tenths of a foot with an average difference of twenty-seven hundredths of a foot. Figure 4.1 shows the final calculated peak water surface elevations with the measured field data, and Figure 4.2 shows the locations of the field sites.

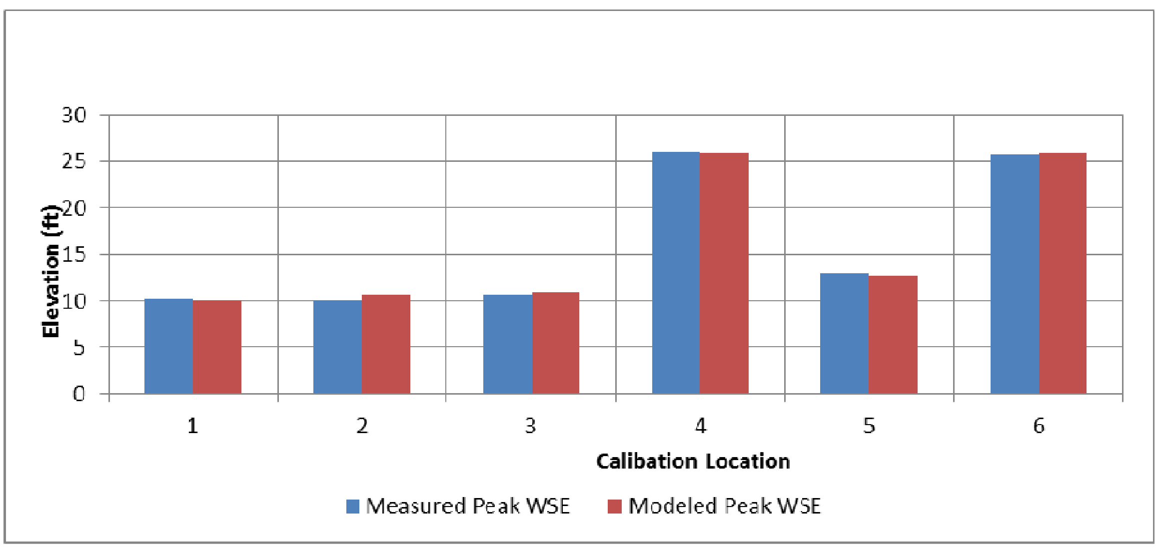

Figure 4.1 Measured vs. Modeled peak water surface elevation (WSE) for the calibration dataset.

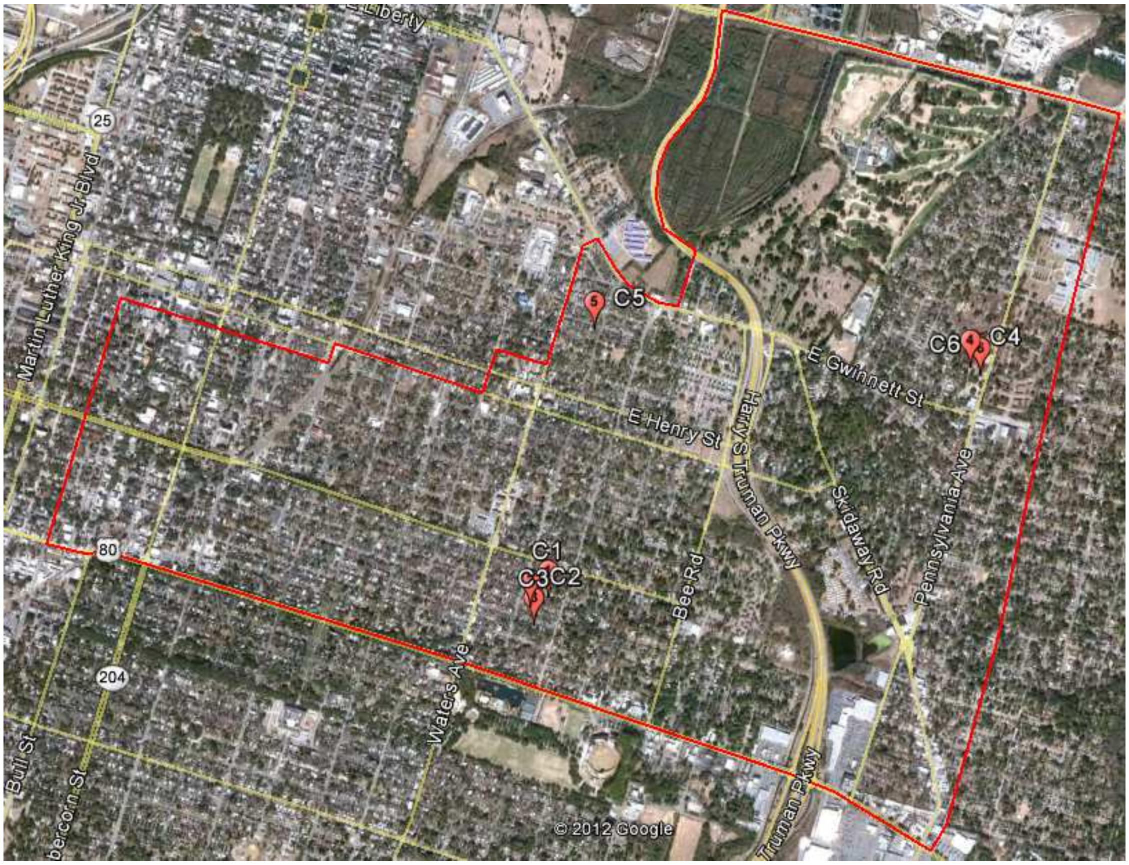

Figure 4.2 Locations where peak water surface elevation was observed and used to calibration the simulation model.

4.1.2 Model Evaluation

Once the calibration was complete, the additional data set was used to evaluate the model. The difference in the measured versus the modeled peak water surface elevations ranged from zero to two feet with an average difference of eighty-seven hundredths of a foot. One important note is that the location where the values differed by two feet is located on the drainage divide of the basin. It is possible and quite likely given the topography of the region that additional storm water runoff could have been received by this basin from the neighboring drainage basin to the south. Figure 4.3 details the relationship between the measured values and the modeled values and Figure 4.4 shows the locations of the field evaluation sites.

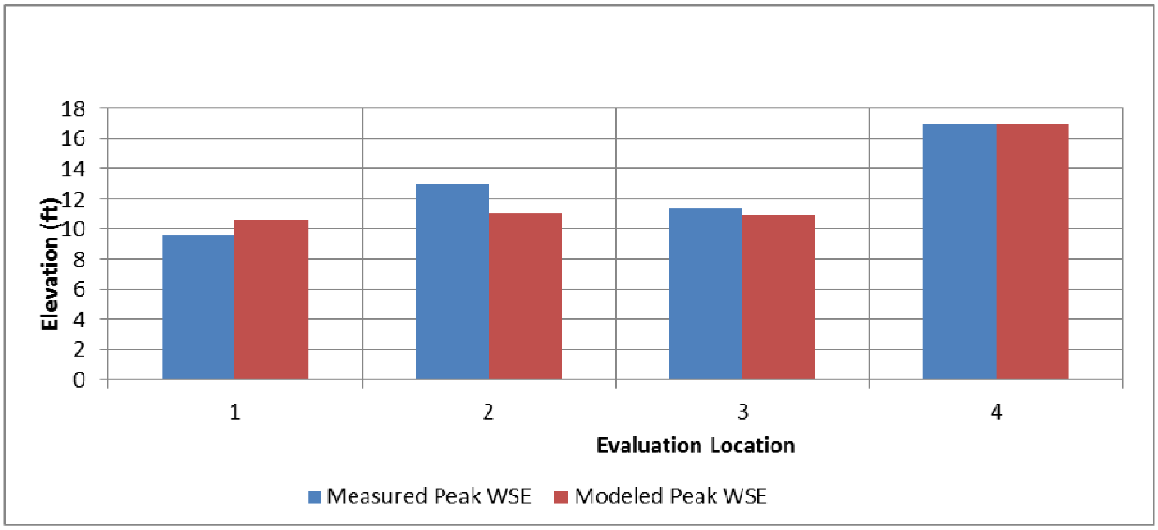

Figure 4.3 Measured vs. modeled peak water surface elevation (WSE) for the evaluation dataset.

Figure 4.4 Locations where peak water surface elevation was observed and used to evaluate the simulation model.

In addition to the field measured elevations, fourteen road intersections were recorded as having flooded during this storm event. While there was no elevation or flooding depth recorded for these intersections, the model was checked to verify if the intersections that were flooded during the rainfall event were flooded during the computer simulation. Of the fourteen intersections, the peak water surface elevation was shown to meet or exceed the ground elevation at the center of the intersection for twelve of the intersections or 86 percent. Figure 4.5 details the relationship between the ground elevation and the calculated peak water surface elevations.

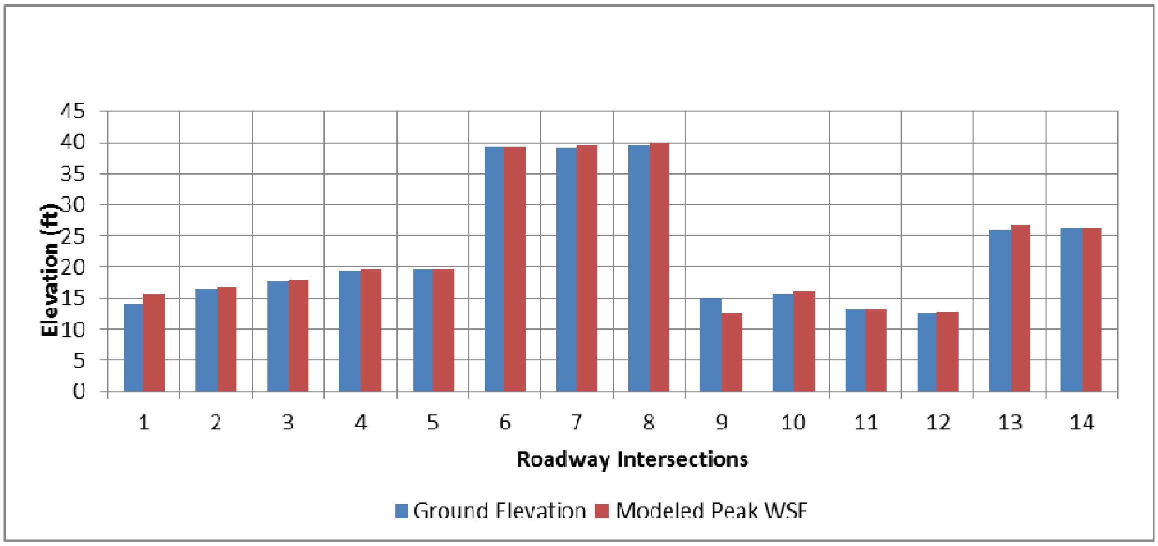

Figure 4.5 Measured vs. modeled peak water surface elevation (WSE) for a dataset of inundated roadway intersections that was not used in the model calibration procedure.

The model also predicted that additional intersections would flood. Intersection flooding in this study was defined as intersections where the peak water surface elevation exceeded the ground elevation by one foot or more. With a flooding depth of less than one foot, it is unlikely that the entire intersection was flooded and subsequently these intersections are unlikely to be reported. According to the model, an additional forty three intersections would experience flooding. Even though this is a significant number of locations, it is reasonable to expect that some of the intersections would not be reported as flooded due to the limited amount of time of inundation. Anecdotally, flooding is not generally reported for roadways and streets while the rainfall event is occurring. Using the time of most intense rainfall (approximately three hundred and thirty minutes) only thirty two intersections are predicted as flooding.

4.2 Discussion

This thesis provides for the creation of a two dimensional floodplain model using XP-SWMM in Savannah, Georgia. With the creation of this model, it was possible to model a recorded rainfall event and calibrate the runoff characteristics to match flooding depths and duration at multiple locations throughout the basin. The end result is a model that can be used as a predictor of flooding for the established one percent annual chance rainfall event. The model can also be modified to assess potential improvements within the watershed and analyze any potential impacts to the peak water surface elevations.

The model created for the Casey Canal North basin was calibrated with a storm that occurred on June 29, 1999. The City of Savannah used a field gage to record the rainfall data for the event and also gathered peak water surface elevations and flooding duration during and after the rainfall event and recorded the information by street addresses.

Using information provided by the City of Savannah, the hydraulic network was constructed. Once the model was constructed, the rainfall data was input into the model and executed. The hydrologic variables, namely curve number and time of concentration, were manipulated to obtain similar results for the modeled event and recorded event. After several iterations, the modeled results appear to closely represent the field gathered data. A complete list of the calculated peak water surface elevations is provided in the Appendix along with the peak flow information for the hydraulic system.

The hydraulic model was constructed using the historical system maps provided by the City of Savannah. Although the City provided valid information concerning pipe diameters, lengths, and overall system connectivity, little is known about construction materials and condition of the pipes. Therefore, no calibration of parameters was performed on the hydraulic portion of the model. This limitation required all calibration to be completed on the hydrologic parameters, notably curve number and time of concentration.

Without calibration of the hydraulic variables, it is possible that the hydrologic parameters were modified to overcome inaccuracy within the hydraulic variables. According to Pappenberger et. al (2005), the Manning's roughness coefficient is said to be the most important factor for forecasting flood inundation. It was also determined in that study that the variances within the roughness coefficients will not have a significant impact on overall modeling results, however, they can considerably impact local results (2005). Using this knowledge, it is reasonable to assume that the localized flooding elevations used to calibrate the model could in fact be caused by variances in the Manning's coefficient, not the hydrologic characteristics as used in this study. Further investigation will be required to validate the material construction and condition within the hydraulic network to further explore this potential limitation of the model.

In order to complete this report, XP-SWMM was chosen to model the intricacies of the basin in a two-dimensional form. XP-SWMM is capable of considering the loss of floodplain storage and the corresponding loss of conveyance whereas a one-dimensional model is not. XP-SWMM is also capable of maintaining a dynamic link between the one-dimensional model and the surface flow model, specifically by using the water surface elevation profile created by the one-dimensional model as a boundary condition for the surface model. This allows for a more accurate prediction of flood wave propagation. One limitation within the TUFLOW module of XP-SWMM is the limited amount of cells available to model the surface flow. The software application used for this study was limited to ten thousand cells. Each cell was roughly ten meters square. Given the size of the basin, it is unlikely that this limitation greatly affected the final modeled results. However, it is possible that a higher level of accuracy with the results could be obtained by increasing the total number of cells.

CHAPTER 5

CONCLUSIONS AND FUTURE WORK

5.1 Conclusions

The primary goal of this thesis was to create a computer model of urban flooding within Savannah, GA, resulting from a storm event that occurred in June 1999. The model was developed using existing information on the topography, stormwater infrastructure, and meteorological data within the Casey Canal North basin in Savannah. A dataset of observed peak water surface elevation levels at different locations within the study area was used to both calibrate and evaluate the accuracy of the model.

The model calibration used a subset of the observed water surface elevation level dataset. Hydrologic model parameters were manually adjusted to match these observed water surface elevation levels. The calibration of the model using the hydrologic parameters of curve number, time of concentration, and drainage area were adjusted within a constrained range that was determined by engineering judgment of the uncertainty of the model parameters. The calibrated model was able to produce similar results as was measured in the field following a rainfall event of June 1999. While there were some differences in the calculated values for peak water surface elevations and the measured peak water surface values, and an automated calibration routine may result in a better calibrated model, the results were deemed acceptable for the purposes of this study.

Upon completion of the model calibration, the model was evaluated against two datasets. First, it was evaluated by the subset of observed water surface elevation measurements not used in the model calibration. Second it was evaluated against a dataset of inundated road crossings to test if the model correctly predicted that the road crossings would be inundated. For the comparison of the calibration dataset, the difference between the measured and modeled results ranged from one-tenth to six-tenths of a foot with an average difference of twenty-seven hundredths of a foot. For the evaluation dataset, the difference between the measured and modeled results ranged from zero to two feet with an average difference of eighty-seven hundredths of a foot. For the roadway intersections where flooding was denoted during the June 1999 rainfall event, 86% of the intersections experienced flooding during the modeling of the storm. These evaluation measures provided confidence that the model is capable of predicting the flooding event.

5.2 Future Work

Since the focus of this thesis was on development, calibration, and evaluation of the urban flooding model for the study area, the model can now be used for various planning activities. For example, one future use of the model can be to model the inundation resulting from a SCS Type III, one percent chance storm with a total rainfall of ten inches. This rainfall event is consistent with the Federal Emergency Management Agency requirements for creation of proposed Flood Insurance Rate Maps (FIRM).

The development of revised Flood Insurance Rate Maps for the City of Savannah will allow the City to delineate potential flood prone areas and analyze potential improvements to the system. Water surface elevation contour maps can be generated using the TUFLOW module within XP-SWMM and transposed over the storm water system and street maps. The water surface elevations can then be compared with the existing topography maps and verified. Based upon the final results, the model provides an accurate representation of the flooding conditions within the Casey Canal North drainage basin. The predicted flood elevations appear reasonable based upon the results seen previously in the field.

Although the model created within this study has its limitations, the predicted results appear reasonable for forecasting potential flooding. As a subsequent project, the City of Savannah intends to make storm drainage improvements within the Casey Canal North Basin based upon the results provided by this study. Therefore, by using a dynamic model with integrated mapping capabilities, the proposed FIRM maps can be updated as needed with relative ease.

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